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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

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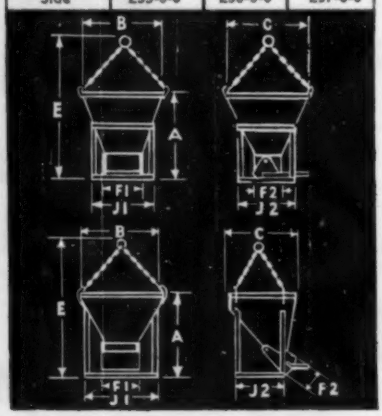

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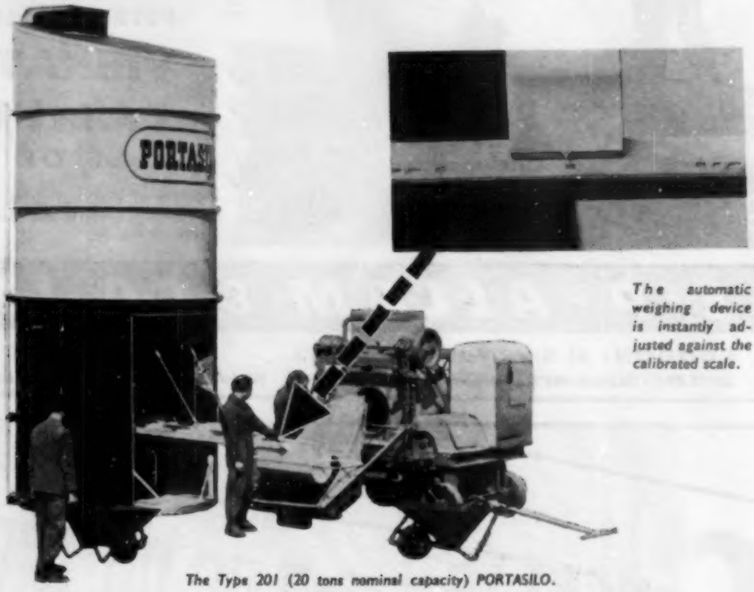

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Volume XLIX, No. 8. LONDON, AUGUST, 1954.

EDITORIAL NOTES

Foundations.

IN few branches of civil engineering is there so much uncertainty or so much left to the judgment of the engineer as in the case of foundations. The new Code of Practice * contains much of our present knowledge of this subject, and its compilers are to be congratulated on giving so many practical suggestions. It is doubtful if many engineers have experience of all types of foundations, or of the type best suited for every problem ; this code describes many of the types of foundation available and helps in selecting the most suitable solution of a problem.

The causes and forms of corrosion of metals are dealt with and it is explained that electrolytic corrosion may be caused when two dissimilar metals are in contact in the presence of moisture. Electrolytic action is similar to that in a galvanic cell ; in the case of zinc-coated steel the zinc acts as the anode and is corroded, the moisture acts as the electrolyte, and the steel acts as the cathode and is protected at the expense of the zinc. This type of corrosion is not as a rule serious, but in metals subject to atmospheric pollution or salt water serious corrosion may occur. In considering the effects of sea-water on reinforced concrete, the need for dense concrete is emphasised. It is well known that sulphate-resisting Portland cement has greater resistance to sea-water than ordinary Portland cement, and that high-alumina cement is practically immune from attack ; it is emphasised, however, that when high-alumina cement is used in large masses of concrete precautions must be taken to dissipate the heat evolved during the setting of the cement. Dense and impermeable concrete is also recommended for resisting the disintegrating effects of peaty waters.

In a section on underwater concreting the code states that better quality concrete can be produced by using colloidal grout in place of ordinary grout. This grout is made by mixing cement, sand, and water in a special mixer, and flows more readily than ordinary grout. Similar grout may be made by adding wetting or dispersing agents to ordinary grout. Colloidal grout can be placed under water without losing its cement content or appreciably absorbing the water in which it is placed. The use of colloidal grout for making concrete is not limited to concrete placed under water.

* Civil Engineering Code of Practice No. 4 (1954). Foundations. (London: Institution of Civil Engineers. Price 15s.)

The code emphasises that exploration of a site and the preliminary design of a foundation must be carried out together. It also draws attention to the fallacy of supposing that a test load on a small area or on a single pile necessarily establishes the bearing capacity of large footings or of groups of piles, and points out that, whereas the bearing capacity of a single footing, pile, pier, or caisson may be ascertained by test loading, this will not indicate the collective strength of the site. Also, settlements recorded with loading tests give no criterion of the probable movement of a full-size structure. In non-cohesive soils the amount of settlement may be estimated from the results of a test load, whereas with clays the influence of time on settlement is so great that such a method of forecasting is useless. In connection with a table classifying the safe bearing capacity of rocks and soils, a warning is given of possible settlements which may necessitate allowable bearing pressures being considerably less than safe bearing pressures, and rules are given which permit, at considerable depths, increases in safe bearing capacities.

The important subject of raft foundations receives scant attention. A page devoted to this type of foundation in the code when it was issued for comment in 1950 stated that the design of raft foundations requires considerable knowledge and care, and that the distribution of bending moments and shears in a raft are not capable of precise determination. It is therefore surprising that, considering the experience that has been accumulated of this type of foundation, no detailed information is given on this subject.

The section on piled foundations contains warnings about spacing piles too closely and on friction adding to pile loads in certain types and conditions of soil, and points out that the downward movement of driven piles caused by the remoulding of soft clay may be avoided by the use of bored piles. It is stated that the bearing capacity can be most accurately ascertained from tests on groups of piles rather than on a single pile. Bearing capacities found from dynamic pile-driving formulæ are unreliable, and it is recommended that an approximate bearing capacity be obtained from Mr. Hiley's formula. The fundamental assumption made in dynamic pile-driving formulæ is that the resistance of the pile to penetration under permanent load has a relationship to its resistance to driving. This may be reasonably accurate for gravels and coarse sands but is not applicable to piles driven in clay, mud, or saturated soils, as during driving the moisture in these soils lubricates the sides of the piles. While the chapter on piled foundations contains much valuable information, there are some important omissions. For example, it is stated that formulæ are not applicable to systems which provide an enlarged base at the foot of the pile. Thus little help is given in estimating the carrying capacity of this type of pile. There is no mention of circular spun-concrete piles, of prestressed concrete piles, or of the various types of foundation formed of tubes and undercutters, or, apart from screw piles, of large cylinders which are screwed into the soil. Well foundations, much used in India, are also not treated. It is a pity that the term "earth pressure" is used in the code to define the resistance of the ground under a foundation; this term should be confined to cases where the earth does in fact exert pressure such as occurs against retaining walls supporting soil.

Indian Students' Hostel, London.

THE accommodation provided at the Indian Students' Union and Hostel (*Fig.* 1) at the junction of Grafton Way and Fitzroy Street, London, W., includes 55 bedrooms, a main lounge, a ladies' lounge, a dining-room for 100 with kitchen, an assembly hall (*Fig.* 3) to seat 324 with stage, dressing-rooms, etc., a library, discussion rooms, a games-room, a sun terrace at fifth-floor level, offices, a warden's office and flat, and a prayer room.

The main part of the structure is supported on two rows of columns. These columns (*Fig.* 4) are circular or rectangular in shape but of the same cross-sectional area, and the cross sections change at different floor levels. Above the first-floor, load-bearing walls are used. The warden's flat projects above the general roof-level, and there is a dome-shaped "shell" over the prayer room.

At one end of the site there were 18 ft. of gravel, and at the other end 23 ft. of gravel, overlying clay. Ground-water was found in a preliminary test at 14 ft. below pavement level. The bottom of the excavation for the auditorium was 17 ft. below ground level, and for the boiler room 21 ft.; it was anticipated that much pumping would be necessary, but water was encountered and pumping was necessary only in the boiler-room basement. Little use could be made of the excellent bearing value of the gravel as excavation depths were very near to or in the clay, and a bearing pressure of 2 tons per square foot was adopted. The ground-water did not contain sufficient quantities of soluble salts to have any deleterious effect on the concrete.

Fig. 1.—Main Elevation.

The floor of the auditorium is 15 ft. 9 in. below ground-level and is designed to

Fig. 2.—Staircase to Auditorium.

Fig. 3.—The Auditorium.

resist the head of water found by test. It is 1 ft. thick and spans between the continuous footing, 3 ft. 6 in. deep, below the inner row of columns and the footings below the six-sided columns (*Fig.* 4) and retaining wall at the rear of the site. The columns on the main elevation are clear of the auditorium and are on an independent strip-footing, except for the two columns in the boiler room whose foundations are incorporated in the floor which is 2 ft. 6 in. thick. On one side of the boiler room there is a retaining wall to preserve the vaults below the pavement for use as coal stores. Because of the shallow beams required, the columns are at 25 ft. centres.

The load-bearing walls of the upper floors (*Fig.* 5) permitted the use of very shallow beams across the internal corridors, so making it possible to have a continuous pipe-duct over the corridors along the length of the building at each floor level. The walls are 7 in. thick and are at 16 ft. centres. Due to the plan, only three of these frames were similar and two frames have only one spandrel wall.

The chief feature of the main staircase from the ground floor to the auditorium below is a reinforced concrete tapered wall, or " fin " (*Fig.* 2) which carries the wall above and the two columns supporting the main staircase. The total bending moment at the base of the tapered wall due to all conditions of load from column No. 424 was 9,410,000 in.-lb., which was partly counterbalanced by a bending moment in the opposite direction from column No. 325. The final bending

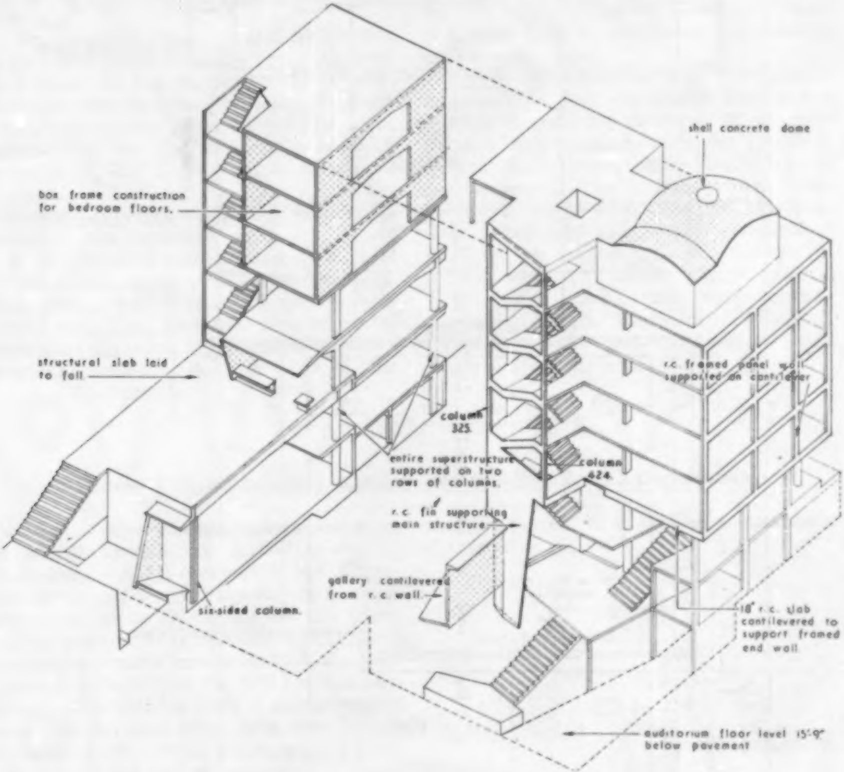


Fig. 4.

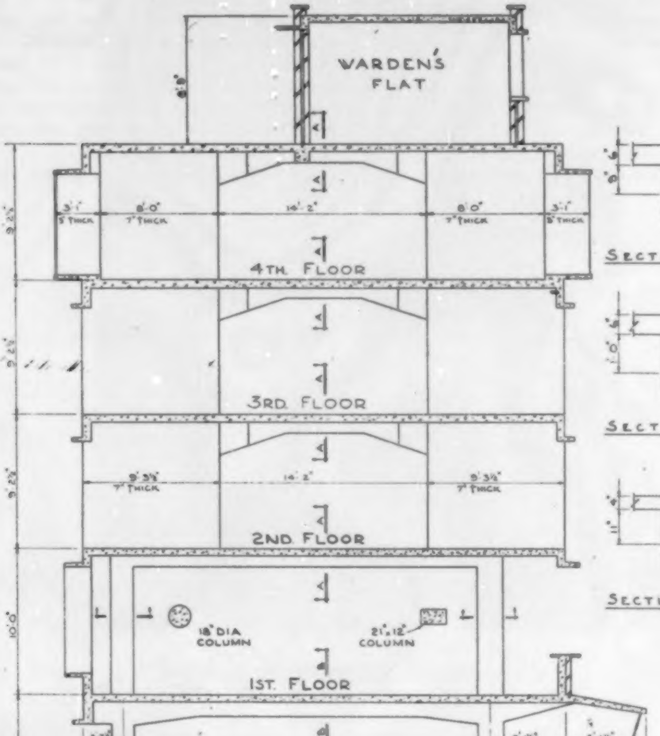


Fig. 5.—Typical Cross Section.

moment therefore was 7,020,500 in.-lb. The maximum compressive stress in the concrete due to this bending moment and the direct load was 1070 lb. per square inch, which was only 70 lb. per square inch more than the permissible design stress. The footing of the tapered wall is 13 ft. 7 in. long by 6 ft. wide by 1 ft. 6 in. thick (that is 6 in. deeper than the auditorium floor of which it is an integral part).

The balcony (*Figs.* 3 and 4) in the auditorium spans from the retaining wall to a beam in the thickness of the slab between the circular columns, beyond which it is cantilevered. Over the main entrance to the auditorium the balcony spans from the bottom of a reinforced concrete wall which in turn spans between the tapered wall and a column as shown in the isometric view.

A cantilevered slab at first-floor level supports the upper part of the building. This slab is 1 ft. 6 in. thick and is reinforced with 1-in. diameter bars at 2½ in. centres at the point of maximum negative bending moment. At the edge of the cantilever is a beam 4 ft. 6 in. deep which distributes the loads from the end frames, so enabling the bending moment on the cantilever to be kept low.

The prayer room on the roof has a dome 3 in. thick of shell construction with a vault 4 in. thick on one side. The thrusts of the dome are resisted on three sides by a beam spanning between the corner columns and on the other side by the vault acting as a beam. There is a glass light in the dome and heating coils in the floor.

The concrete mixture was 1 : 2 : 4 with aggregate of ¾-in. maximum size, and a minimum crushing strength at 28 days of 3000 lb. per square inch was specified. Where there was much congestion of reinforcement the concrete was consolidated with a poker-type vibrator. Test cubes had an average strength of 4840 lb. per square inch at 28 days.

The architect is Mr. Ralph Tubbs, O.B.E., F.R.I.B.A., and the consulting engineers Messrs. Frederick S. Snow & Partners. The main contractors were Messrs. Tersons, Ltd.

Precast Concrete Units applied to Bridge Pier Construction.

The use of precast units to form the piers of the Richmond-San Rafael bridge in California, U.S.A., is reported in "Engineering News-Record" for March 4, 1954. The units serve as moulds for the piers.

The method of construction (*Fig.* 1) is to place a precast base and drive the piles through holes left in the grid for this purpose. The bottom shell is then lowered into position and filled with concrete to a depth of 5 ft. Next the truncated cone, which has been cast on a barge and towed to the site, is lowered into position ; this is followed by the shaft from which reinforcement bars project to bond with the reinforcement in the diaphragm, which is formed between steel shutters. The assembled units are filled with concrete to form a solid pier.

The cones, which are lowered into position in pairs, weigh up to 52½ tons each and the largest of them has a maximum diameter of 34 ft.

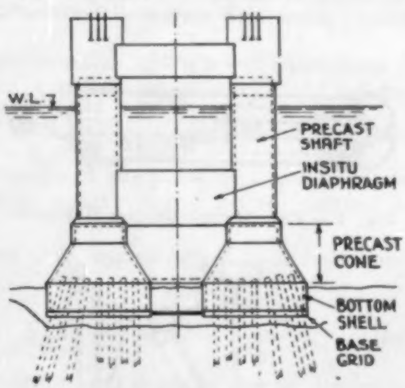


Fig. 1.

Impact Methods of Testing Concrete.

In Report No. 107 (1952) of the German Select Committee for Reinforced Concrete Dr. K. Gaede describes two methods of determining the compressive strength of concrete by non-destructive means analogous to the Brinell method of testing the hardness of metals.

In the first method a spring-loaded hammer (*Fig.* 1), known as the " Frank " hammer, is used and the steel ball [(10) in *Fig.* 1], held in the nut (9), is placed against the surface of the concrete. A steady pressure on the cap (1) drives the spindle (7) into the cylinder against the resistance of a spring (15). The spring presses against the ram (18), which is held to the spindle by claws (6 and 17). When the spindle has been pushed a certain distance into the cylinder one of the claws is released and the ram is driven along the spindle by the spring to strike a collar which transmits the blow to the steel ball. One claw is shorter than the other so that blows of different magnitudes may be given to the ball; claw (6) corresponds to a contraction of the spring of 5 cm. and a blow of 50 kg.-cm. (known as a full blow), claw (17) corresponds to a contraction of the spring of 2·5 cm. and a blow of 25 kg.-cm.

The hammer is about 1 ft. 2 in. long and weighs about 5½ lb. The instrument is calibrated for use when held horizontally. When used vertically the blow is affected by gravity and the force of the hammer is altered as follows. Blow directed vertically upwards: full blow, − 5 per cent.; half-blow, − 10 per cent.; blow directed vertically downwards: full blow, + 5 per cent.; half-blow, + 10 per cent.

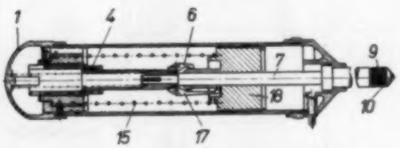


Fig. 1.—The Spring-loaded Hammer.

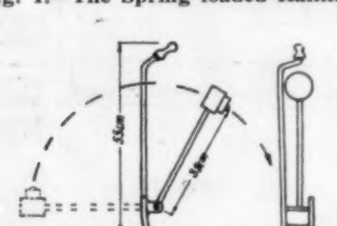


Fig. 2.—The Pendulum Hammer.

A pendulum hammer (*Fig.* 2) is used in the second method. This consists of a handle and a 2-kg. (4·4-lb.) weight at the end of a lever 35 cm. (1 ft. 2 in.) long. If the hammer is allowed to fall through 180 deg. the blow is equivalent to 137 kg.-cm., and if it falls through 90 deg. the blow is equivalent to 68·5 kg.-cm. The instrument is simple, but can be used only on vertical surfaces at positions at least 80 cm. (2 ft. 7½ in.) below its point of support.

The imprints made on the surface of the material under test by either of the hammers are measured by a lens with a magnification of about 6 and provided with scales divided to 0·1 mm.

In making a test not fewer than twenty imprints are made, the hammer being held perpendicular to the face of the concrete to be tested. The imprints should be evenly spaced and obvious faults, pockets, and grains of coarse aggregate avoided. The imprints should be at least about 1½ in. from an edge. The diameters of the imprints are measured to within 0·1 mm. in two directions at a right-angle. Imprints for which these measurements differ by 20 per cent. or more, and irregular imprints, are disregarded. The imprints are usually made with a 10-mm. ball and an impact of 50 kg.-cm., and the diameters should be between 3 mm. and 7 mm. From the mean value of the diameters of a group of imprints the compressive strength of the concrete may be obtained from tables calculated from the results obtained from 600 tests carried out by several laboratories.

The mathematical derivation is given for the relation between the diameter of an imprint and the crushing strength of the concrete. Statistical analyses of the results are necessary because of the large variations between the results of individual tests.

[This method of testing was referred to in the Editorial Notes of this journal for May, 1952.]

Analysis of Statically-indeterminate Structures by the Deformation Method.—II.*

By M. SMOLIRA, Ph.D., A.M.I.C.E., D.I.C.

Single-span Frames with Curved Members.

General Case.—Frames with curved members can be analysed by the deformation method if, in addition to the angular deformations, the displacements at joints due to the curvature of the members are taken into account. As previously, the structure is assumed to be relaxed from the effect of continuity by cutting it at suitable places, load functions and elastic constants are calculated, and equations of equilibrium of each point at which the imaginary cut has been introduced are set out.

The load functions for a curved beam of any shape and cross sections (*Fig.* 14) consist of angular deformations θ_a and θ_b, and also of a linear deformation Δ_o.

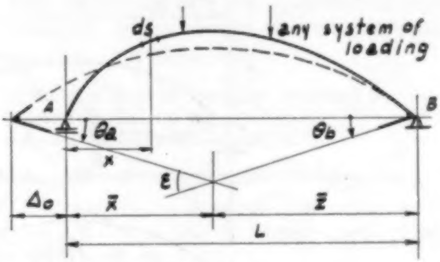


Fig. 14.

These, similarly to equation (2), are calculated from the well-known equations

$$\varepsilon = \int_A^B \frac{mds}{EI}\,;\ \bar{x} = \frac{1}{\varepsilon}\int_A^B \frac{mxds}{EI}\,;\ \bar{z} = L - \bar{x}\,;\ \theta_a = \varepsilon\frac{\bar{z}}{L}\,;\ \theta_b = \varepsilon\frac{\bar{x}}{L}\,;\ \Delta_o = \int_A^B \frac{myds}{EI} \qquad (23)$$

in which ε represents the total angular deformation of a curved beam from A to B, θ_a and θ_b are angular deformations at A and B respectively, $\bar{x}$ and $\bar{z}$ define the centre of gravity of the $\frac{m}{EI}$ diagram, Δ_o is the total horizontal deformation of a curved beam, and x and y are co-ordinates of any point on a beam. Some more complicated cases are calculated from equations (23) by the method of summation.

Examples.—Consider a pitched beam (*Fig.* 15) subjected to a uniformly-distributed load. From (23),

$$EI\theta_a = EI\theta_b = \int_A^B mds = \frac{1}{12}wL^2s\,;\ EI\Delta_o = \int_A^B myds = \frac{5}{48}wL^2ps \quad . \qquad (24)$$

in which s is the length of the inclined portion and p is the rise of the beam.

Equations (23) may also be used to calculate the elastic constants of curved beams. Unit bending moment at the end A (*Fig.* 16*a*) will produce the angular deformations α_{ab} and β, and also linear deformation Δ^m_{ab}. Unsymmetrical curved

* *Continued from July number.*

beams will have different values of α and Δ^m when a unit bending moment is applied at the other end (*Fig.* 16*b*), the values of β being the same according to Clerk Maxwell's Reciprocal Theorem. The angular deformations at A and B due to a unit horizontal force applied along AB (*Fig.* 16*c*) are denoted by γ and δ respectively, and the horizontal translation of the joints by Δ^h. It should be noted that, from the Reciprocal Theorem, γ and δ will be numerically equal to the corresponding values of Δ^m.

As an example of elastic constants of a curved beam, consider a symmetrical pitched beam (*Fig.* 17). Elastic constants, α, β, and Δ^m for the unit bending moment, calculated from (23), are

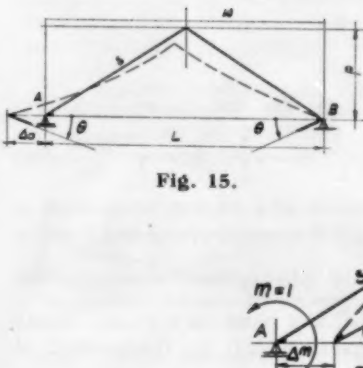


Fig. 15.

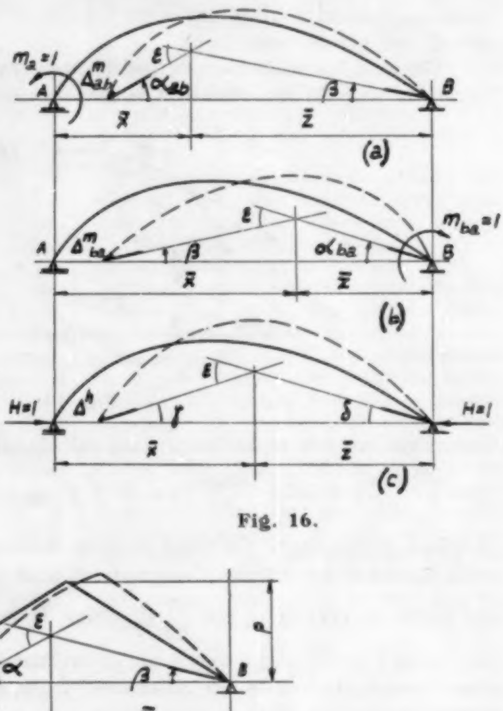


Fig. 16.

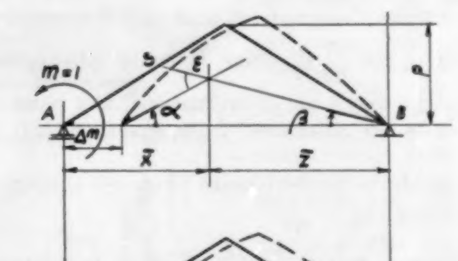


Fig. 17.

$$EI\varepsilon = \int_A^B m.ds = s\,;\ \bar{x} = \frac{L}{3};\ EI\alpha = \tfrac{2}{3}s\,;\ EI\beta = \tfrac{1}{3}s\,;\ EI\Delta^m = \tfrac{1}{2}ps \qquad (25)$$

and the elastic constants due to unit horizontal force are

$$EI\gamma = EI\delta = \tfrac{1}{2}ps\,;\ EI\Delta^h = \tfrac{2}{3}p^2s \quad . \quad . \quad . \quad (26)$$

Symmetrical Frame with Symmetrical Load.—With the load functions and elastic constants calculated from (23), statically-indeterminate bending moments can now be obtained by setting out the equations of equilibrium of angular deformations of the joints. For a symmetrical frame of any shape with symmetrical loading (*Fig.* 18) the angular gaps at A and B are $\left(\theta_o + \tfrac{1}{2}.\frac{\Delta_o}{h}\right)$. The equation of equilibrium for joint A can be set out as follows:

$$\text{CAB, } m_a\alpha_{ab} + m_a\beta + m_a\alpha_{ac} + m_a\frac{\Delta^m}{h} + H\gamma + \tfrac{1}{2}H\frac{\Delta^h}{h} = \theta_o + \tfrac{1}{2}.\frac{\Delta_o}{h} \qquad (27)$$

It should be noted that, in setting out the equations of equilibrium, each figure has its geometrical meaning, which should be well understood and visualised to avoid error. For equation (27) these are

$m_a\alpha_{ab}$, rotation of beam AB due to bending moment m_a at A.
$m_a\beta$, rotation of beam AB due to m_a at B.
$m_a\alpha_{ac}$, rotation of column AC due to m_a at A.
$m_a\frac{\Delta^m}{h}$, rotation of beam AB due to horizontal movement of joint A when m_a is at A.
$H\gamma$, rotation of beam AB due to horizontal force at A.
$\tfrac{1}{2}.H\frac{\Delta_0}{h}$, rotation of beam AB due to horizontal movement of joint A when horizontal force H is applied at A.

For a symmetrical pitched frame (*Fig.* 18) with uniformly-distributed load, the load functions from (24) are $EI_b\theta_o = \frac{1}{12}wL^2s$ and $EI_b\Delta_o = \frac{5}{48}wL^2ps$, and the elastic constants from (25) are $EI_b\alpha = \frac{2}{3}s$, $EI_b\beta = \frac{1}{3}s$, $EI_b\Delta^m = \frac{1}{2}ps$ and $EI_b\gamma = EI_b\delta = \frac{1}{2}ps$, $EI_b\Delta^h = \frac{2}{3}p^2s$. Substituting these values in (27),

$$m_a\left(\frac{2s}{3I_b} + \frac{s}{3I_b} + \frac{h}{3I_c} + \frac{ps}{2hI_b} + \frac{ps}{2hI_b} + \frac{p^2s}{3h^2I_b}\right) = \frac{wL^2s}{12I_b} + \frac{5}{96}\cdot\frac{wL^2ps}{hI_b} \qquad (28)$$

and, if $I_b = I_c$,
$$m_a = \frac{wL^2}{32}\cdot\frac{hs(8h + 5p)}{h^3 + 3h^2s + 3hps + p^2s} \quad . \quad . \quad . \quad (29)$$

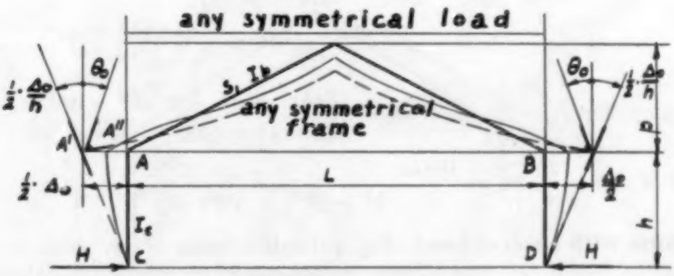


Fig. 18.

General Case of Frame with Curved Member.—In a general case of a frame with a curved member, of any shape and with any system of loading (*Fig.* 19), the angular gaps at A and B are $\left(\theta_a + \frac{\Delta a}{h}\right)$ and $\left(\theta_b + \frac{\Delta_o - \Delta a}{h}\right)$, in which Δa represents the final horizontal translation of joint A ; other symbols are as defined previously. It should be noted that point A′ in *Fig.* 19 represents

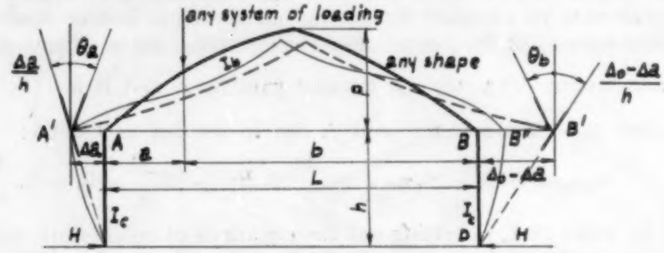


Fig. 19.

the final position of joint A, while point B′ will move under the action of moments m_a and m_b and force H before reaching its final position at B″. The values of m_a, m_b, and Δa are calculated for two conditions of angular deformations at A and B, and for one condition of shear, as follows :

$$\left.\begin{aligned}
&\text{CAB, } m_a\alpha_{ab} + m_b\beta + m_a\alpha_{ae} + H\gamma = \theta_a + \frac{\Delta a}{h}\\
&\text{ABD, } m_a\beta + m_a\frac{\Delta^{ma}}{h} + m_b\alpha_{ba} + m_b\alpha_{bd} + m_b\frac{\Delta^{mb}}{h} + H\delta + H\frac{\Delta^h}{h} = \theta_b + \frac{\Delta_o - \Delta a}{h}\\
&\text{S.c., } \quad \frac{m_a}{h} = \frac{m_b}{h}.
\end{aligned}\right\} \quad (30)$$

For a pitched frame with a concentrated load W the elastic constants are given by equation (25), and equations (30) become

$$\left.\begin{aligned}
&m_a\left(\frac{2s}{3I_b} + \frac{s}{3I_b} + \frac{h}{3I_c} + \frac{ps}{2hI_b}\right) = \frac{Wabs(L+b)}{3L^2I_b} + \frac{\Delta a}{h}E\\
&m_a\left(\frac{2s}{3I_b} + \frac{s}{3I_b} + \frac{ps}{2hI_b} + \frac{h}{3I_c} + \frac{ps}{2hI_b} + \frac{ps}{2hI_b} + \frac{2p^2s}{3h^2I_b}\right)\\
&\qquad = \frac{Wabs(L+a)}{3L^2I_b} + \frac{Waps(3L^2 - 4a^2)}{6hL^2I_b} - \frac{\Delta a}{h}E
\end{aligned}\right\} \quad (31)$$

from which, for $I_b = I_c$, $m_a = m_b = \frac{Wahs}{4L^2} \cdot \frac{6bhL + p(3L^2 - 4a^2)}{h^3 + 3h^2s + 3hps + p^2s}$. (32*a*)

and, for $a = b = \frac{L}{2}$, $m_a = \frac{WhLs}{8} \cdot \frac{3h + 2p}{h^3 + 3h^2s + 3hps + p^2s}$ . . . (32*b*)

Frame with Lateral Load (*Fig.* 20).—If a frame of any shape is subjected to any lateral load acting on a column, equations (30) require only slight adjustment as follows :

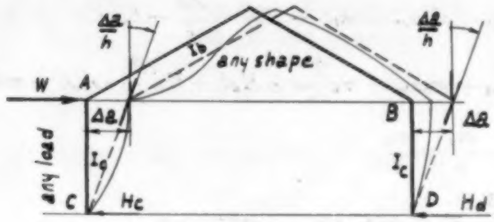


Fig. 20.

$$\left.\begin{aligned}
&\text{CAB,}\quad m_a\alpha_{ab} + m_a\alpha_{ac} - m_b\beta - H_d\gamma = \frac{\Delta a}{h}\\
&\text{ABD,}\quad m_b\alpha_{ba} + m_b\alpha_{bd} + m_b\frac{\Delta^{mb}}{h} + H_d\delta + H_d\frac{\Delta^{h}}{h} - m_a\beta - m_a\frac{\Delta^{ma}}{h} = \frac{\Delta a}{h}\\
&\text{S.c.,}\quad \frac{m_a + m_b}{h} = W_a
\end{aligned}\right\} \quad (33)$$

For a symmetrical pitched frame with a horizontal force acting at A (*Fig.* 20), equations (33) become

$$\left.\begin{aligned}
&\text{CAB,}\quad m_a\frac{2s}{3I_b} + m_a\frac{h}{3I_c} - m_b\frac{s}{3I_b} - m_b\frac{ps}{2hI_b} = \frac{\Delta a}{h}E\\
&\text{ABD,}\quad m_b\frac{2s}{3I_b} + m_b\frac{h}{3I_c} + m_b\frac{ps}{2hI_b} + m_b\frac{ps}{2hI_b} + m_b\frac{2p^2s}{3h^2I_b} - m_a\frac{s}{3I_b} - m_a\frac{ps}{2hI_b} = \frac{\Delta a}{h}E\\
&\text{S.c.,}\quad m_a + m_b = Wh
\end{aligned}\right\} \quad (34)$$

from which, if $I_c = I_b$,

$$m_a = \frac{Wh^2}{4}\cdot\frac{2h^2 + 6hs + 3ps}{h^3 + 3h^2s + 3phs + p^2s}\,; \quad m_b = \frac{Wh}{4}\cdot\frac{2h^3 + 6h^2s + 9hps + 4p^2s}{h^3 + 3h^2s + 3hps + p^2s} \quad (35)$$

Frame with a Crane Load (*Fig.* 21).—If a frame with a curved member is subjected to a crane load, the angular gaps at A and B are $\left(\frac{\Delta a}{h} - \theta_{ac}\right)$ and $\frac{\Delta a}{h}$ respectively, and equations (33) require only slight adjustment :

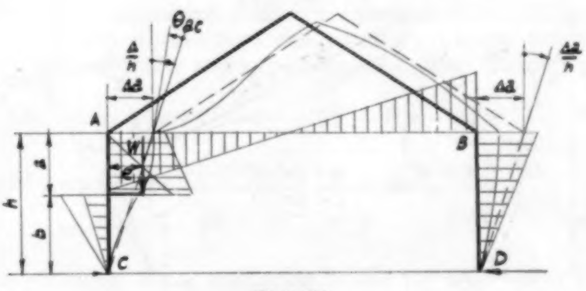


Fig. 21.

$$\left.\begin{aligned}
&\text{CAB,}\quad m_a\alpha_{ab} + m_a\alpha_{ac} - m_b\beta - H_d\gamma = \frac{\Delta a}{h} - \theta_{ac}\\
&\text{ABD,}\quad m_b\alpha_{ba} + m_b\alpha_{bd} + m_b\frac{\Delta^{mb}}{h} + H_d\delta + H_d\frac{\Delta^h}{h} - m_a\beta - ma\frac{\Delta^{ma}}{h} = \frac{\Delta a}{h}\\
&\text{S.c.,}\quad \frac{m_o}{h} - \frac{m_a}{h} = \frac{m_b}{h}
\end{aligned}\right\} \quad (36)$$

in which $m_o = We$.

For a symmetrical pitched frame (*Fig.* 21),

$$\left.\begin{aligned}
&\text{CAB,}\quad m_a\frac{2s}{3I} + m_a\frac{h}{3I} - m_b\frac{s}{3I} - m_b\frac{ps}{2hI} = \frac{\Delta}{h}E - \theta_{ac}\\
&\text{ABD,}\quad m_b\frac{2s}{3I} + m_b\frac{h}{3I} + m_b\frac{ps}{2hI} + m_b\frac{ps}{2hI} + m_b\frac{2p^2s}{3h^2I} - m_a\frac{s}{3I} - m_a\frac{ps}{2hI} = \frac{\Delta a}{h}E.\\
&\text{S.c.,}\quad m_a + m_b = We
\end{aligned}\right\} \quad (37)$$

in which $$EI\theta_{ac} = \frac{We}{6h^2}(a^3 + 3a^2b - 2b^3).$$

North-light Frame.—The load functions for a north-light frame (*Fig.* 22) are calculated as follows. For a uniformly-distributed load $w = 1000$ lb. per ft.,

$$EI\varepsilon = \frac{1 \times 24^2}{12 \times 30}[3 \times 6 \times 13{\cdot}42 + 26{\cdot}84(30 + 2 \times 6)] = 2190.$$

$$\bar{x} = \frac{4 \times 6^2 \times 13{\cdot}42 + 26{\cdot}84(8 \times 30^2 - 11 \times 24 \times 30 + 4 \times 24^2)}{2[3 \times 6 \times 13{\cdot}42 + 26{\cdot}84(30 + 3 \times 6)]} = 18.$$

$$EI\theta_a = 2190 \times \frac{12}{30} = 876\,;\quad EI\theta_b = 1314.$$

$$EI\Delta_0 = \frac{24^2 \times 12}{24 \times 30}[4 \times 6 \times 13{\cdot}42 + 26{\cdot}84(5 \times 30 - 4 \times 24)] = 17{,}000.$$

The elastic constants are :

For $m_{ab} = 1$,

$$EI\varepsilon = \frac{1}{2 \times 30}[13{\cdot}42(30 + 24) + 26{\cdot}84 \times 24] = 22{\cdot}81.$$

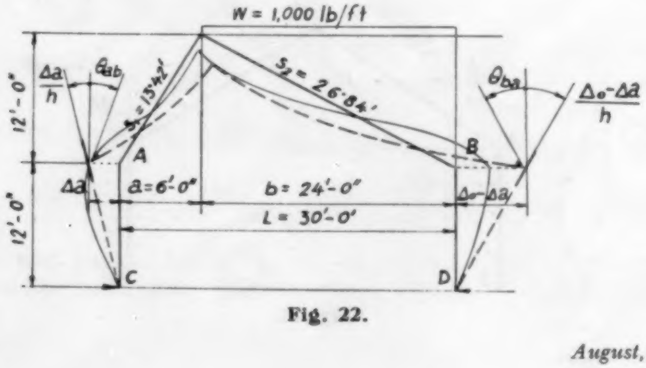


Fig. 22.

$$\bar{x} = \frac{1}{3} \times \frac{13\cdot42 \times 6(30 + 48) + 26\cdot84 \times 24(30 + 12)}{13\cdot42(30 + 24) + 26\cdot84 \times 24} = 8\cdot12 \text{ ft.}$$

$$EI\alpha_{ab} = 22\cdot81 \times \frac{21\cdot88}{30} = 16\cdot64\,;\quad EI\beta = 6\cdot17.$$

$$EI\Delta^{ma} = \frac{12}{6 \times 30}[13\cdot42(30 + 48) + 2 \times 26\cdot84 \times 24] = 155\cdot67.$$

For $m_{ba} = 1$,

$$EI\varepsilon = \frac{1}{2 \times 30}[26\cdot84(30 + 6) + 13\cdot42 \times 6] = 17\cdot44.$$

$$\bar{z} = \frac{1}{3} \times \frac{26\cdot84 \times 24(30 + 12) + 13\cdot42 \times 6 \times (30 + 48)}{26\cdot84(30 + 6) + 13\cdot42 \times 6} = 10\cdot62.$$

$$EI\alpha_{ba} = 11\cdot27\,;\quad EI\beta = 6\cdot17.$$

$$EI\Delta^{mb} = \frac{12}{6 \times 30}[26\cdot84(30 + 12) + 2 \times 13\cdot42 \times 6] = 85\cdot8.$$

For $H = 1$, $EI\gamma = \Delta^{ma} = 155\cdot67\,;\ EI\delta = \Delta^{mb} = 85\cdot8\,;$

$$EI\Delta^{h} = \frac{12^2}{3}(13\cdot42 + 26\cdot84) = 1932.$$

Substituting these values in (30),

$$\text{CAB,}\quad m_a\left(16\cdot64 + 6\cdot17 + \frac{12}{3} + \frac{155\cdot67}{12}\right) = 876 + \frac{\Delta a}{h}EI.$$

$$\text{ABD,}\quad m_a\left(11\cdot27 + 6\cdot17 + \frac{12}{3}\quad \frac{155\cdot67}{12} + \frac{85\cdot8}{12} + \frac{85\cdot8}{12} + \frac{1932}{12^2}\right) = 1314 + \frac{17{,}000}{12} - \frac{\Delta a}{h}EI$$

from which $m_a = 35{,}620$ ft.-lb.

Frames with Ties.—Frames with curved beams and joined with ties (*Fig.* 23) can be analysed in a similar way to frames previously discussed except that, in addition to the equations of angular deformations, the equations of horizontal translations must be taken into account. In a statically-determinate condition, with joints A and B relaxed from the effect of continuity, the tie is assumed to be detached at B. The angular and linear gaps formed in this condition are as shown in *Fig.* 23. Statically-indeterminate bending moments and force P in the tie are calculated from two equations of equilibrium of angular deformations at A and B, and from one equation of linear deformation at B. These, for any shape of frame and any system of loading, are set out as follows :

$$\left.\begin{aligned}
&\text{CAB,}\quad m_a\alpha_{ab} + m_a\alpha_{ac} + m_b\beta + (H + P)\gamma = \theta_a + \frac{\Delta a}{h}\\
&\text{ABD,}\quad m_b\alpha_{ba} + m_b\alpha_{bd} + m_a\beta + (H + P)\delta = \theta_b + \frac{\Delta t - \Delta a}{h}\\
&\text{B,}\qquad m_a\Delta^{ma} + m_b\Delta^{mb} + (H + P)\Delta^{h} = \Delta_0 - \Delta t
\end{aligned}\right\} \quad . \qquad (38)$$

$\Delta t = \dfrac{PL}{E_sA_s}$, in which Δt is the elongation of the tie under force P, A_s and E_s

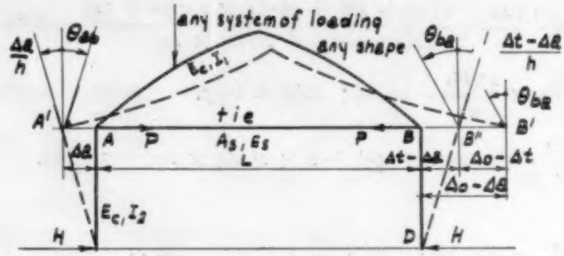


Fig. 23.

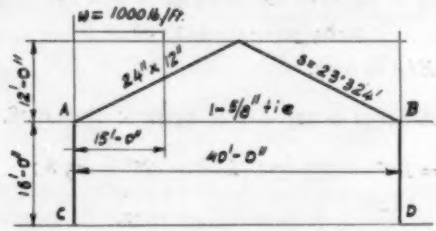


Fig. 24.

are the area of cross section of the tie and its modulus of elasticity respectively, and other symbols are as defined previously.

For a symmetrical pitched roof with any system of loading, equations (38) become

$$\text{CAB,}\quad m_a\left(\frac{2s}{3E_cI_1}+\frac{h}{3E_cI_2}+\frac{s}{3E_cI_1}+\frac{ps}{2hE_cI_1}\right)+P\frac{ps}{2E_cI_1}=\frac{\theta_a}{E_cI_1}+\frac{\Delta a}{h}.$$

$$\text{ABD,}\quad m_a\left(\frac{s}{3E_cI_1}+\frac{2s}{3E_cI_1}+\frac{h}{3E_cI_2}+\frac{ps}{2hE_cI_1}\right)+P\frac{ps}{2E_cI_1}=\frac{\theta_b}{E_cI_1}+\frac{\Delta t}{h}-\frac{\Delta a}{h}.$$

$$\text{B,}\quad m_a\left(\frac{ps}{2hE_cI_1}+\frac{ps}{2hE_cI_1}+\frac{2p^2s}{3h^2E_cI_1}\right)+P\frac{2p^2s}{3hE_cI_1}=\frac{\Delta}{hE_cI_1}-\frac{\Delta t}{h}.$$

or, eliminating Δa,

$$\left.\begin{aligned} m_a\left(2s+\frac{2h}{3}.\frac{I_1}{I_2}+\frac{ps}{h}\right)+P\left(ps-\frac{LI_1}{nA_sh}\right)&=\theta_a+\theta_b\\ m_a\left(\frac{ps}{h}+\frac{2p^2s}{3h^2}\right)+P\left(\frac{2p^2s}{3h}+\frac{LI_1}{nAh}\right)&=\frac{\Delta_0}{h}\end{aligned}\right\}\quad . \qquad (39)$$

in which $n=\dfrac{E_s}{E_c}$.

As a numerical example, consider a pitched frame (*Fig.* 24) with uniformly-distributed load on part of the beam, for which $I = 0{\cdot}666$ ft.^{4}, $A_s = 0{\cdot}307$ sq. in. (0·00216 sq. ft.), and $n = 15$. The load functions, calculated from equations (24) to (26) are $EI\theta_a = 1154{\cdot}81$; $EI\theta_b = 813{\cdot}15$; $EI\Delta_0 = 14{,}267{\cdot}73$, and (39) become

$$m_a\left(2 \times 23{\cdot}324 + \frac{2 \times 16}{3} + \frac{12 \times 23{\cdot}324}{16}\right)$$
$$+ P\left(12 \times 23{\cdot}324 - \frac{40 \times 0{\cdot}666}{15 \times 0{\cdot}00216 \times 16}\right) = 1154{\cdot}81 + 813{\cdot}15.$$
$$m_a\left(\frac{12 \times 23{\cdot}324}{16} + \frac{2 \times 12^2 \times 23{\cdot}324}{3 \times 16^2}\right)$$
$$+ P\left(\frac{2 \times 12^2 \times 23{\cdot}324}{3 \times 16} + \frac{40 \times 0{\cdot}666}{15 \times 0{\cdot}00216 \times 16}\right) = \frac{14{,}267{\cdot}73}{16},$$

from which $m_a = 20{,}777$ ft.-lb. and $P = 1811$ lb.

Influence of Change of Temperature and Settlement of Supports on Frames with Curved Members.

CHANGE OF TEMPERATURE.—The influence of change of temperature on frames with curved members, of any shape (*Fig.* 25), can be calculated from the general equations (30), which require only slight modification as follows:

$$\left.\begin{aligned}
&\text{CAB},\ m_a\alpha_{ab} + m_a\alpha_{ac} + m_a\beta + H\gamma = \frac{\Delta a}{h_1}.\\
&\text{ABC},\ m_b\alpha_{ba} + m_b\alpha_{bd} + m_a\beta + m_a\frac{\Delta^{ma}}{h_2} + m_b\frac{\Delta^{mb}}{h_2} + H\delta + H\frac{\Delta^h}{h_2} = \pm\frac{\Delta t - \Delta a}{h_2}\\
&\text{S.c.},\ \frac{m_a}{h_1} = \frac{m_b}{h_2}.
\end{aligned}\right\} \quad (40)$$

in which Δt is the total deformation of the curved member along the line AB due to change of temperature, and other symbols are as defined previously.

For symmetrical frames of any shape, equations (40) reduce to a form similar to (27):

$$\text{A},\ m_a\alpha_{ab} + m_a\alpha_{ac} + m_b\beta + m_a\frac{\Delta^m}{h} + H\gamma + \tfrac{1}{2}H\frac{\Delta^h}{h} = \pm\frac{\Delta t}{2h} \quad . \quad (41)$$

For symmetrical pitched frames, equation (41) becomes

$$\text{A},\ m_a\left(\frac{2s}{3EI_b} + \frac{h}{3EI_c} + \frac{s}{3EI_b} + \frac{ps}{2hEI_b} + \frac{ps}{2hEI_b} + \frac{p^2s}{3h^2EI_b}\right) = \pm\frac{\Delta t}{2h}.$$

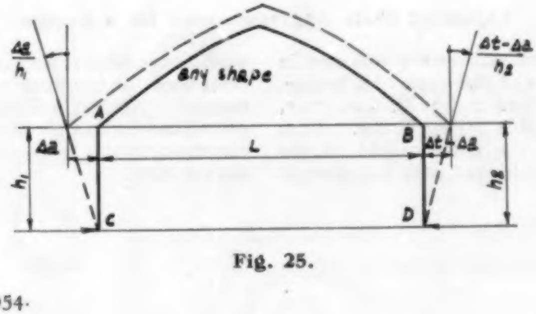


Fig. 25.

Substituting in this equation,

$$\Delta t = 2\int_A^e \alpha_t T ds \cos\phi = \alpha_t TL \quad . \quad . \quad . \quad . \quad (42)$$

we obtain, for $I_b = I_c$, $m_a = \dfrac{3h\Delta tEI}{h^3 + 3h^2s + 3hps + p^2s}$ . . . . . (43)

in which α_t is the coefficient of thermal expansion, T is the change of temperature, ϕ the angle between the inclined member of the frame and a horizontal, and other symbols are as defined elsewhere.

HORIZONTAL MOVEMENT OF SUPPORTS.—The influence of horizontal movement of supports can be analysed in a similar way. The angular gaps at A and

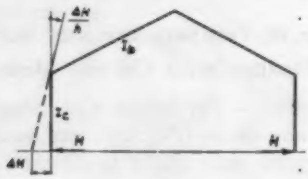


Fig. 26.

B (*Fig.* 26) are $\dfrac{\Delta H}{h}$ and zero respectively, and the equation of equilibrium becomes

$$\text{A,}\quad m_a\alpha_{ab} + m_a\beta + m_a\alpha_{ae} + m\frac{\Delta^m}{h} + H\gamma + H\tfrac{1}{2}\frac{\Delta^h}{h} = \tfrac{1}{2}\frac{\Delta H}{h} \quad . \quad (44)$$

For a symmetrical pitched roof,

$$m_a\left(\frac{2s}{3EI_b} + \frac{s}{3EI_b} + \frac{h}{3EI_c} + \frac{ps}{2hEI_b} + \frac{ps}{2hEI_b} + \frac{p^2s}{3h^2EI_b}\right) = \frac{\Delta H}{h},$$

from which, for $I_b = I_c$,

$$m_a = \frac{3h\Delta HEI}{h^3 + 3h^2s + 3hps + p^2s} \quad . \quad . \quad . \quad (45)$$

(*To be continued.*)

Expanded Shale Aggregate used for a Bridge.

EXPANDED-SHALE AGGREGATE was used in the construction of the deck of a bridge, nearly 3900 ft. long and 31 ft. 6 in. wide, over the Columbia river, U.S.A. It is estimated that the dead weight of the structure is about 1340 tons less than it would have been if normal aggregates had been used for the deck. The strength of concrete cylinders was more than 3000 lb. per square inch at 28 days and at 45 days the strength was in excess of 5000 lb. per square inch.

Partially-prestressed Flat Slabs.

THE greatest stresses in flat-slab floors or roofs are near the heads of the columns, due to the maximum bending moments which occur at the heads of the columns and the shearing forces around the column capitals. The bending moments necessitate a grid of large diameter bars in the top of the slab, the thickness of which is frequently determined by the shearing stresses. In general the shearing forces must be resisted by the concrete alone, otherwise the reinforcement over the column capital would be too congested to allow the proper placing of the concrete. These high stresses are, however, localised and affect an area of only 10 to 15 per cent. of the total area of the slab.

In order to avoid these difficulties without increasing the size of the capitals a method has been devised and patented by a Belgian engineer, M. A. Lipski, in which prestressed and reinforced concrete is used over the columns in the parts of the slabs where there would otherwise be much congestion of reinforcement. The greater part of the floor or roof is of reinforced concrete slabs which are bonded to the prestressed slabs by mild steel bars projecting from the prestressed slabs. The slabs over the column heads are therefore reinforced slabs which are also prestressed and are designed as reinforced concrete members subjected to combined bending and compression.

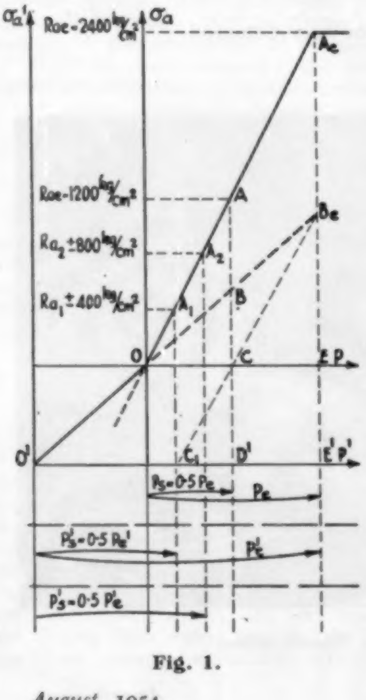


Fig. 1.

The stress in the mild steel in the prestressed slab must, however, be less than that in an ordinary reinforced concrete slab if the ratio between the stress in the steel under load and the elastic limit is not to be reduced. Consider *Fig.* 1 in which σ_a is the stress in the steel due to the applied load on the slab, R_a the allowable stress, and P the applied load. The line OA_e shows the increase in stress as a function of the load assuming that the concrete cannot resist tensile forces, and OB_e shows the same relationship on the assumption that the concrete is prestressed and can therefore resist tension. Suppose, for example, that the elastic limit of the steel (R_{ae}) is 2400 kg. per sq. cm., and that the allowable stress in tension in the steel (R_a) is 0·5 R_{ae}, that is 1200 kg. per sq. cm.; this stress is chosen both on account of the quality of the steel and of the necessity to limit cracking of the concrete. Under the load P_s causing a stress R_a, and from *Fig.* 1,

$$\frac{P_s}{P_e} = \frac{OC}{OE} = 0{\cdot}5,$$

where P_e is the load causing a stress in the steel equal to the elastic limit. If a prestressing force is applied to the unloaded slab the origin of the curves will be moved from O to O′, where O′ is on a continuation of OB_e. It will be seen that, if the steel is then stressed to 1200 kg. per sq. cm.,

$$\frac{P'_s}{P'_e} = \frac{O'D'}{O'E'} > 0{\cdot}5,$$

and that the factor of safety with respect to the elastic limit of the steel is less than in the previous case. In fact, to obtain the same factor of safety the stress must be reduced to 400 kg. per sq. cm. so that

$$\frac{P'_s}{P'_e} = \frac{O'C'}{O'E'} = 0{\cdot}5.$$

An analogous reasoning may be used

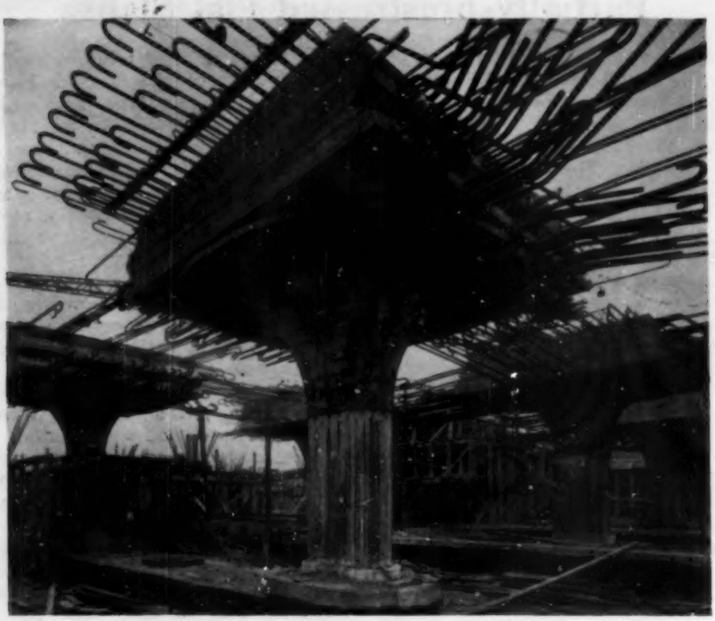


Fig. 2.—Prestressed Slabs over Columns.

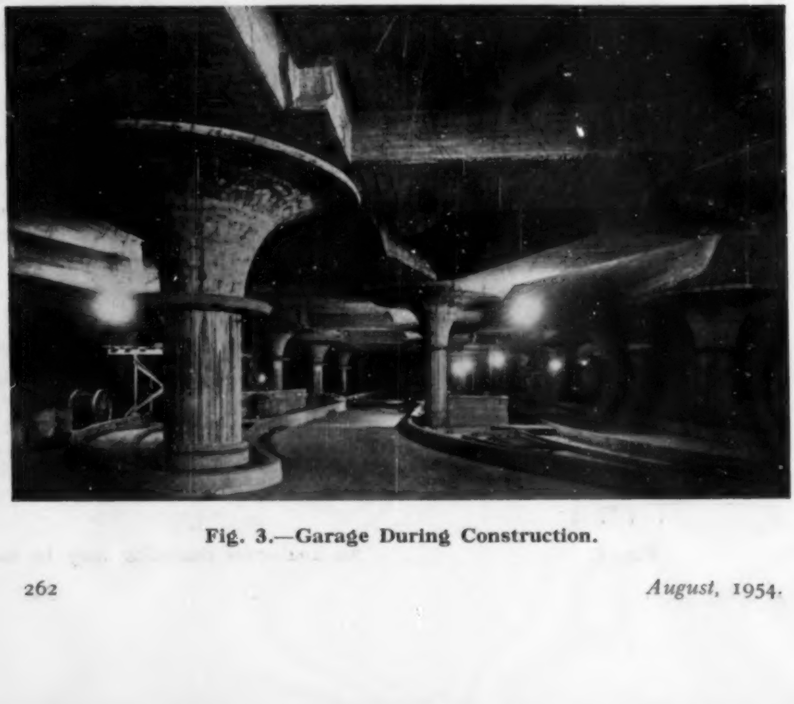


Fig. 3.—Garage During Construction.

Fig. 4.—Anchor for Cable.

with regard to the factor of safety against cracking of the concrete, but in general it is not necessary to make this calculation unless high-tensile steel is used.

The stress of 1200 kg. per sq. cm. is generally chosen only as an indirect method of preventing cracking, and it is common in Belgium to use in structural steel a stress R_a of 1400 kg. per sq. cm. (about 0·6 R_{ae}) in which case a stress of 800 kg. per sq. cm. may be used in the case of the partially-prestressed slab to which *Fig.* 1 relates.

In the design of such a prestressed slab there are obviously many solutions, depending on the proportion of prestressing steel to reinforcement; the least number of prestressing wires is determined by the need to reduce the shearing forces so that these may be resisted by the concrete alone.

A Structure in Brussels.

The method described in the foregoing has been applied in the construction in Brussels of a bus station the roof of which supports the railway lines joining the Bruxelles–Nord and Bruxelles–Midi railway stations. The bus station is 398 ft. long by 106 ft. wide at one end and 148 ft. wide at the opposite end, and has a total area of about 6700 sq. yd. The roof is supported on 57 columns at 35-ft. centres in one direction and 30-ft. centres in the other direction. These columns are of ordinary reinforced concrete and are 3 ft. 9 in. diameter with a capital 15 ft. 10 in. diameter. Because of the heavy load (1640 lb. per square foot) for which it was designed, the roof comprises prestressed concrete slabs about 16 ft. square by 4 ft. thick (*Fig.* 2) over the columns, reinforced concrete beams 4 ft. deep by 13 ft. wide between the columns, and a reinforced concrete slab 1 ft. 4 in. thick between the beams. The use of shallow beams for the column-strips of the flat slab was required by the heavy load. The partly completed garage is shown in *Fig.* 3. The Belgian method of prestressing was used. The cables were looped and the wires exposed at one end (*Fig.* 4) to form an anchor, the other end (to which the jack was applied) being anchored by wedges and sandwich-plates.

A design entirely in reinforced concrete was originally prepared but was abandoned because of the economy of the partially-prestressed slabs. The quantities of materials required for the two schemes are as follows, those for the reinforced concrete slab being given first. Concrete (cu. yd.): 8900, 6100; Reinforcement (tons): 2250, 1135 (including 15 tons of high-tensile steel for the cables). The total reduction in cost of the building due to the use of the prestressed slab is between 25 and 30 per cent. of the cost of the reinforced concrete building.

The architects were MM. Fernard Petit and Yvan Blomme, the consulting engineer was M. A. Lipski, and the contractor was F. M. Gillion.

The Effect of Wind on the Watertightness of High Walls.

In a paper presented to the recent conference of the Royal Institute of British Architects, Mr. W. A. Allen and Mr. Edward D. Mills said that the tall United Nations building in New York was exposed to high winds and driving rain. The walls are entirely of glass and are therefore non-absorbent. It was found that during high winds and storms the rain blown against the building travelled upwards and entered weep-holes and openings that had been designed on the assumption that the water would flow downwardly. It had been necessary to carry out extensive repairs and to seal vulnerable places and joints. The authors pointed out that in tall and exposed structures with walls of non-absorbent materials the construction must be completely watertight.

The Use of Fly Ash in Concrete.

The use of fly ash in concrete has received much attention in recent years in the U.S.A., where it has been used as an admixture and also as a replacement for a portion of the cement. Benefits have been achieved in certain circumstances. Fly ash is the residue from the burning of powdered coal collected from the flue gases of coal-burning power plants. Generally it is very fine, approaching or exceeding the fineness of Portland cement, and consists principally of the oxides of silicon, aluminium, and iron with varying amounts of carbon. Since it consists of extremely fine, nearly spherical, particles, fly ash may increase the workability of concrete; this is important in lean mixtures or in concrete made with aggregates deficient in fines. The finely-divided silica in certain fly ashes prevents the expansion which sometimes results from chemical reactions between cement and aggregates. The silica in fly ash, as in other pozzolanas, has the ability to combine with the lime liberated during the hydration of cement to form cementitious compounds; this reaction takes place slowly over long periods, thus contributing to the development of the strength of concrete, chiefly at ages of several months or longer.

The results of tests on fly ash in concrete are given in a paper by Mr. Delmar L. Bloem issued by the National Ready Mixed Concrete Association of the U.S.A. The conclusions reached as a result of these tests are as follows:

(1) The effect of fly ash is related in a general way to its fineness, carbon content, and silica content. These factors alone, however, are not enough to provide a quantitative indication of its quality. Materials which conform to the generally-accepted limits for these properties may affect concrete quite differently, particularly in the development of strength.

(2) In some cases mixing water may be reduced slightly for a given consistency, and workability and surface appearance may be improved in the case of lean concretes or where aggregates are deficient in fines.

(3) Fly ash of suitable chemical composition and fineness increases the strength of concrete at later ages. The increase of strength is relatively small at early ages but becomes greater with increasing age. The greatest benefits are to very lean concrete, possibly because the fly ash increases workability.

(4) Almost without exception the replacement of 10 per cent. or more of cement with fly ash results in reductions in strength at seven and 28 days. At three months and one year strength equal to or greater than that of normal concrete may be obtained with some fly ashes when as much as 1 cu. ft. of cement is replaced in a cubic yard of concrete.

(5) Low temperature or lack of adequate moisture during curing appears to be no more detrimental to fly ash concrete than to normal concrete, although the advantage of increased strengths at later ages is lost unless moist curing is continuous.

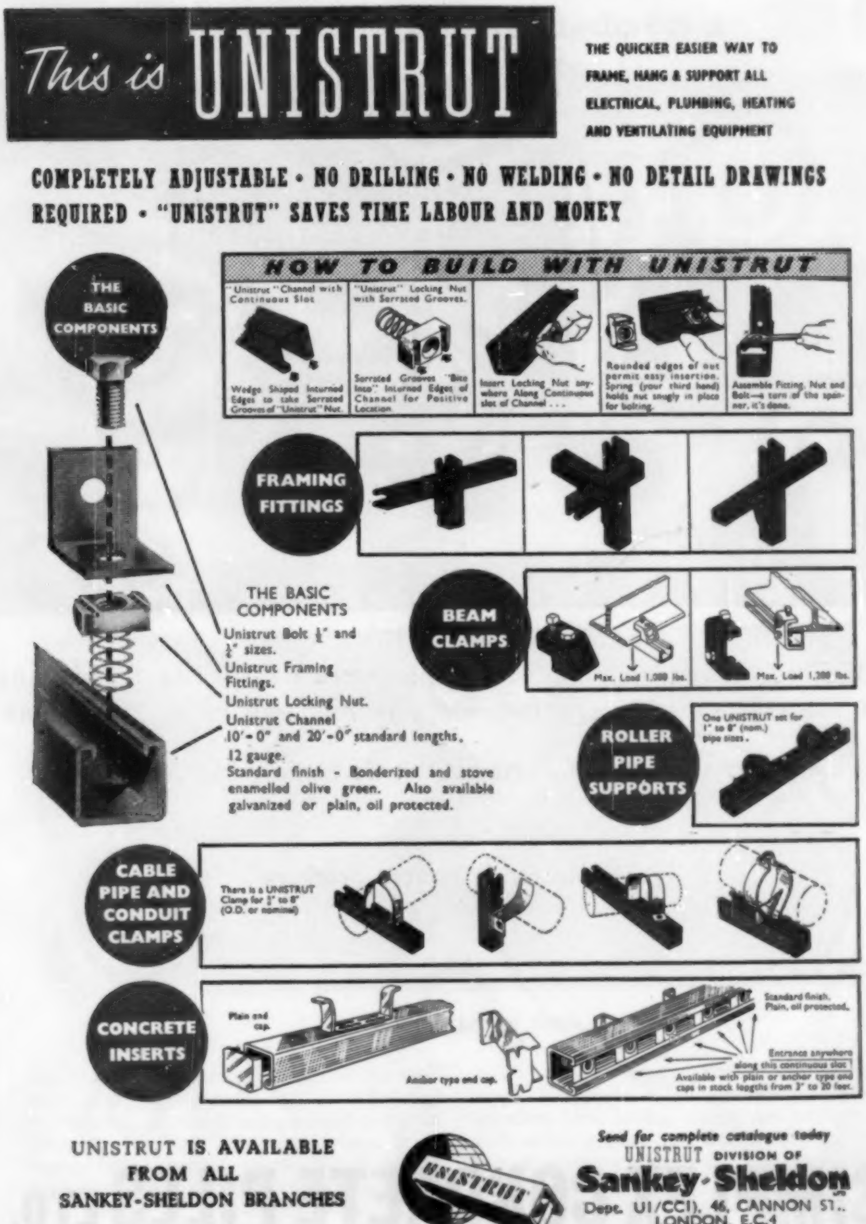
(6) The amount of air-entraining agent required to produce a given air content may be several times as great for concrete made with fly ash as for normal concrete.

(7) It does not appear that fly ash is significantly beneficial or detrimental to the freezing and thawing resistance of concrete, although evidence is inconclusive.

(8) Certain fly ashes improve the resistance of concrete to sulphate, and tend to prevent expansion due to reaction between the cement and alkalis in the aggregate.

(9) Shrinkage of concrete does not appear to be significantly affected by fly ash.

(10) Due to its formation of cementitious compounds over long periods, fly ash may reduce the permeability of concrete at later ages.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Blastfurnace Cement Made on a Construction Site.

THE "Trief" process of making Portland blastfurnace cement is being used by Mitchell Engineering, Ltd., in the construction of two dams near Glen Moriston, Scotland. This process, which is a Belgian invention, was used in the construction of a large dam at Bort-les-Orgues, France. The two dams in Scotland will contain more than 300,000 cu. yd. of concrete, and the arrangement of the concrete plant is shown in *Fig.* 1. The slag is delivered by road from a steelworks at Glasgow to Glen Moriston, where it is tipped into a hopper (on the left of *Fig.* 1) from which it is taken by conveyors either to a hopper of 50 tons capacity at the millhouse or to storage. The maximum daily consumption of the mills (*Fig.* 2) is about 170 tons.

In the millhouse a rotary-plate feeder delivers the slag to a conveyor-belt, which has a scale on it to weigh the material before it enters the mill. There are two mills, each about 5 ft. diameter by 28 ft. long, and each is driven by a 275-h.p. motor. Each mill can grind to the required fineness 3½ to 3¾ tons of slag an hour. The grinding process is a wet one, and the slurry has a water content of about 30 per cent. When the slurry leaves the mills it passes along sloping launders into tanks, which are provided with stirring gear and compressed air to keep the slurry agitated. The tanks are 14 ft. in diameter by 13 ft. high, and are about 20 ft. lower than the level of the mills.

Due to the nature of the site the batching plant (*Fig.* 3) is distant from the mills, and the slurry is pumped through steel

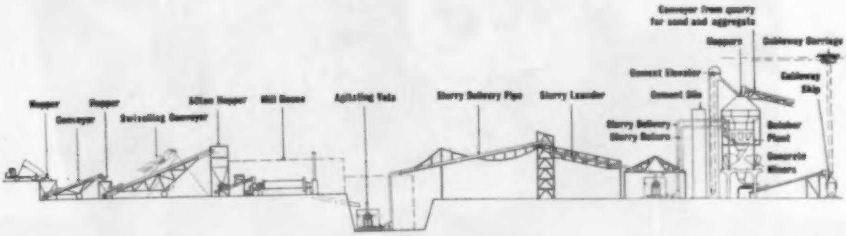


Fig. 1.—Arrangement of Plant.

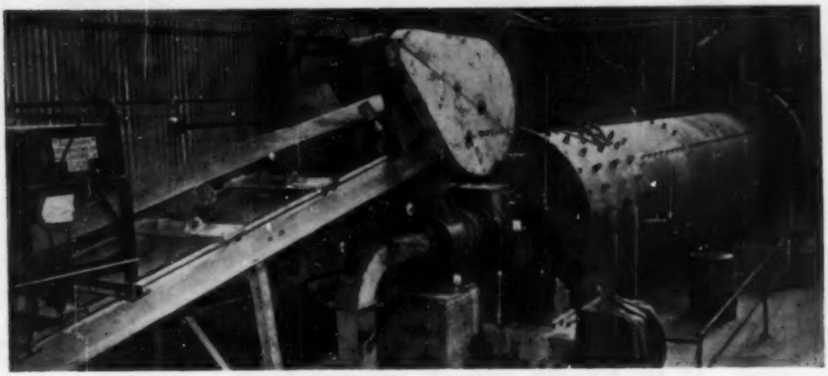


Fig. 2.—One of the Mills.

pipes for a distance of about 200 ft. To keep the pipes clean when the pumps are not working a rubber ball is blown through them. At the end of the pipes the slurry is delivered to tanks near the batching plant, where it is again kept agitated. These tanks, together with the tanks at the mill, provide storage for about 200 tons of slurry. The slurry is pumped to a small tank at the top of the batching plant; to prevent settlement, a return is taken from the bottom of the upper tank to the lower tank so that the slurry is kept circulating. All the materials required for the concrete (the slurry, Portland cement, sand, and coarse aggregate) are weighed in the required proportions and delivered to the mixer. The concrete is taken by conveyors to hoppers which supply the skips that are carried by cableways to the work.

Although concrete made with this cement takes longer to harden, it has less heat of hydration than Portland cement and is thus advantageous in large masses of concrete. The process dispenses with the need to grind together the slag and Portland cement clinker as is the case with ordinary Portland blastfurnace cement. If necessary the slurry may be dried and stored.

A result of a typical test showed that a mixture of 1 part of Trief cement to 7 parts of aggregate had the following properties: Density at seven days, 151·9 lb. per cubic foot; compressive strength, 1540 lb. per square inch at three days and 2742 lb. per square inch at seven days. This concrete comprised 1207 lb. of sand, 570 lb. of aggregate from 3/16 in. to 3/4 in., 635 lb. of aggregate from 3/4 in. to 1 1/2 in., 44 1/2 lb. of aggregate from 1 1/2 in. to 2 1/4 in., 318 lb. of aggregate from 2 1/4 in. to 3 1/2 in., 99 lb. of Portland cement, 348 lb. of slurry, 243 lb. of water (including the water in the slurry), and 5 1/2 lb. of accelerator. All the aggregates, including the sand, were crushed granite.

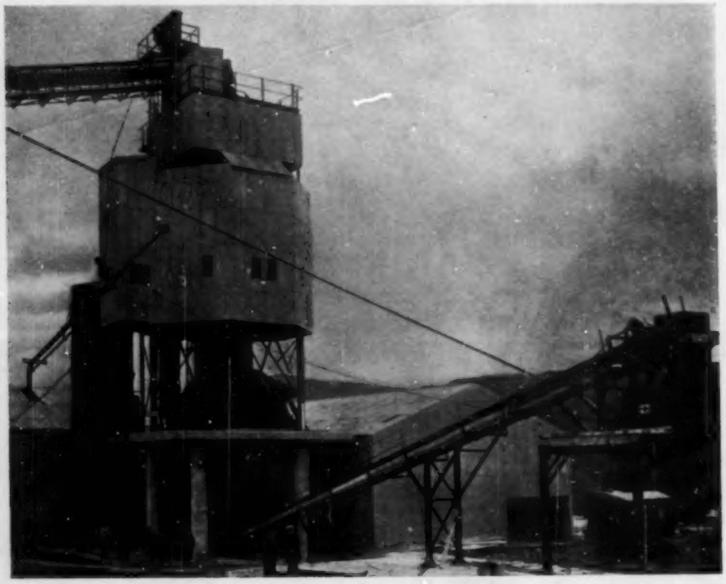
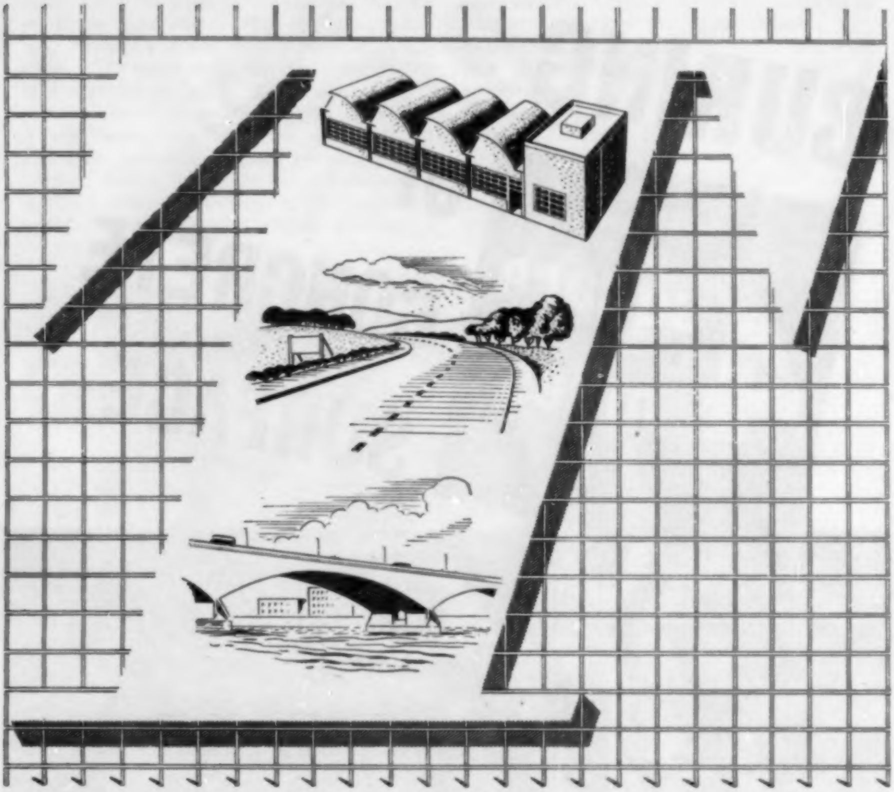


Fig. 3.—Concrete Mixing Plant.



Expansion and Contraction Joints in Roads.

An article in a recent number of "Engineering News-Record", describes some concrete roads recently constructed by the New Jersey (U.S.A.) State Highway Department. In the northern part of the State, where the greatest density of traffic occurs, residual and glacial soils predominate which have the property of "pumping" at joints or cracks even under fairly light traffic. As a result, roads in southern New Jersey are, in general, now constructed on a granular sub-base 8 in. thick, and in northern New Jersey the sub-base is usually 12 in. thick consisting of a fairly clean granular material with a low clay content.

The possibilities of using plain concrete were considered about ten years ago and slabs 60 ft. long, 12 ft. wide, and 8 in. thick were laid to determine what provisions should be made to ensure that cracks would occur only at the contraction joints and that such cracks would occur at all joints at an early stage in the life of the road. Early cracking was considered to be desirable for two reasons: (1) To prevent excessive permanent opening of the joints at which cracks first occur due to infiltration into the open joints; (2) To ensure that the cracks would develop around the large aggregate—since, as the strength of the concrete increases, the cracks tend to pass directly through the larger aggregate, with resulting loss of interlock.

These studies also showed that contraction joints must be closely spaced if the opening is to be small enough to avoid complete loss of interlocking between the pieces of aggregate.

Expansion Joints 105 ft. Apart.

The first road built to test these conclusions was laid during the latter part of 1942 and early in 1943. The road is 10 in. thick on a compacted granular sub-base 12 in. thick. Expansion joints were spaced at 105-ft. intervals, and did not contain dowels. Contraction joints, also without dowels, were spaced 15 ft. apart. Wooden fillets were placed in all expansion joints, and at the joints the ends of the slabs were supported on plain concrete sills 4 in. thick. The sills were 5 ft. wide (2 ft. 6 in. on each side of the joint) and in most cases of a length equal to the

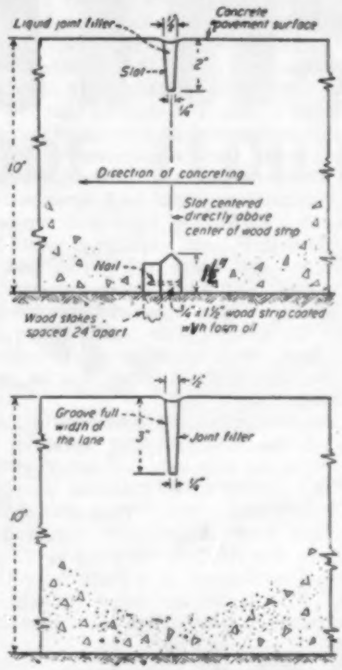


Fig. 1.—Details of the Contraction Joints.

width of the carriageway. The transfer of load at the contraction joints was entirely dependent upon the interlocking of the aggregate. In the contraction joints a strip of wood, ¾ in. thick by 1½ in. high, was placed transversely on the sub-grade, and a groove, 2 in. deep, was formed in the slab directly over the wooden strip. A section through the joint is shown in the upper part of *Fig.* 1.

Gauge-plugs were fixed near many of the expansion and contraction joints to determine the changes in width of the joint. The first measurements were taken as soon as the concrete had hardened sufficiently to hold the plugs securely in position. Measurements taken at intervals during ten years show that there has been a progressive and permanent closure of the expansion joints, and a corresponding increase in the width of the contraction joints. This action took place during

the early life of the road and now appears to have stopped, as the wood in the expansion joints now resists further compression. This condition has been found to be characteristic of later work.

For concrete placed in November, 1942, measurements in January, 1951 (slab temperature 37 deg. F.), showed that the expansion joints were an average of 0·212 in. less in width than when constructed and that the average width of the intermediate contraction joints had increased by 0·122 in. In August, 1951 (slab temperature 94 deg. F.), the expansion joints had closed by an average of 0·303 in. and the contraction joints had opened an average of 0·06 in. These measurements showed that if the opening of the contraction joints is to be limited, expansion joints should not be provided, at least not at close intervals.

Examination of the contraction joints showed that the strips of wood in the bottom of the slab were not entirely effective in causing early cracking at all of the contraction joints. It is stated that although some faults have occurred at joints in this road, at no point is the differential settlement at a joint more than ¼ in. The faults are more pronounced at expansion joints than at contraction joints.

Expansion Joints 240 ft. and 343 ft. Apart.

The second plain concrete road was cast during 1946. This was also 10 in. thick with similar contraction joints, and expansion joints filled with 1 in. of cork but without dowels. Contraction joints were spaced at intervals of 17 ft. 2 in. so that every other one would be in line with expansion joints in an existing adjacent road. Expansion joints at 240-ft. intervals were placed in a length of 2400 ft. of this road and at 343-ft. intervals for a length of 3400 ft. Slabs 3400 ft. and 2400 ft. long were also laid without expansion joints.

On this road there was a rapid closing of the expansion joints and a corresponding opening of the contraction joints. The low frictional resistance of the sand base, 8 in. thick, on which the slab was laid may have contributed to this, but three other causes are believed to have been of greater importance. They were : (1) The road slab was constructed during the spring. As a result its temperature during construction was lower than the temperature it attained at midsummer of the same year. Consequently, the road slab expanded within a few weeks after construction. (2) The expansion joints were widely spaced (240 ft. and 343 ft.), and consequently a small unit expansion resulted in considerable movement at the joints. (3) The expansion-joint filler consisted of cork, ½ in. thick, which offered little resistance to compression.

A third road of this type was built during 1949. It comprises a 10-in. thick slab on a 12-in. sub-base of granular material, which in turn was placed on 6 in. of selected material. The contraction joints are formed by a groove 3½ in. deep on the surface and the wooden strips at the bottom of the joints were omitted because they were not sufficiently effective in causing early cracking at the contraction joints. Also, considerable difficulty had been experienced previously in forming the grooves in the surface directly over the wooden strips. The contraction joints are about 0·08 in. wide in cold weather and during hot weather they close to 0·015 in. There have been no cracks between contraction joints, and hardly any faults are apparent at the joints despite heavy traffic. These joints are shown in the lower part of *Fig.* 1.

Expansion Joints Two Miles Apart.

The latest road being constructed has a more stable sub-base on which the plain concrete slab is being laid. The slab is 10 in. thick, without expansion joints, and with contraction joints, without dowels, at 15-ft. intervals. Because the work consists of additions to, and the re-siting of, an existing road, the longest slab without expansion joints will be a little over 2 miles in the total length of 4·6 miles of new work. A cross section through the road is shown in *Fig.* 2.

The sub-base comprises two layers, of which the material in the upper layer, 4 in. thick, is graded so that it passes a 4-in. sieve, and no more than 50 per cent. will pass a ⅜-in. sieve. The elutriable content is limited to a maximum of 12 per cent. This layer is of broken stone containing sufficient fine stone and screenings to fill completely all voids. The material in the lower 8 in. of the sub-base is gravel containing not more than 5 per cent. of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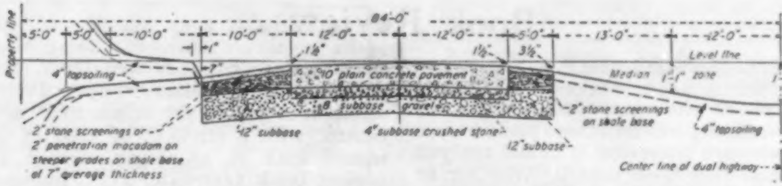

Fig. 2.—Cross Section through Road with Expansion Joints Two Miles Apart.

material removable by elutriation. This course is spread across the entire width of the road, under both the shoulders and and carriageway, and compacted to a depth of 8 in. The top layer is spread across the width of the carriageway and rolled. Compaction is by not less than five passes of pneumatic-tyred rollers or other equipment developing a wheel load of not less than 8 tons on a width of tyre of not more than 2 ft. This has been done because measurements taken on the earlier plain concrete roads indicate that contraction joints even at 15-ft. intervals open during the winter to such an extent as to make it doubtful whether there is effective transfer of the load. As previously stated, these joints will be open about 0·08 in. during cold weather. Because of this and the heavy traffic on this road, the object has been to increase the sub-base so that it will be capable of preventing faults even if there is no transfer of load from one slab to the next.

At day's work joints, a stopboard is set between the shutters about 7½ ft. from the last contraction joint. Through holes at 12-in. intervals along the centre of this board are placed ¾-in. diameter deformed bars, 3 ft. long, which project for half their length into the fresh concrete. When work is started the next day the board is slipped off the projecting ends of the bars before concrete is placed. The high bond strength of the deformed bars checks any tendency for the joint to open and the bars also serve to transfer load across the joint.

This work is being carried out under the direction of Mr. Ransford J. Abbott, State Highway Commissioner, by the Grandview Construction Corporation.

[Regarding the present condition of these roads, Mr. van Breemen, Supervising Engineer, New Jersey State Highway Department, writes: "All the roads are in very good condition. The only defects of any consequence are confined to the joints where differential settlement has occurred in a small number of cases in the roads constructed more than five years ago. The likelihood of cracking is considered to be of less concern than a progressive increase in differential settlement across the joints and the possibility of failure in compression in the concrete due to the entry of solid material in the joints. The difference in temperature during the year, measured at the mid-depth of the slab, is ordinarily from 20 deg. F. to 110 deg. F. It is considered necessary for heavy traffic either that all the joints should contain dowels or that the slab should be cast on a sub-grade of high load-bearing capacity, as the opening of the joints in cold weather is so large that transfer of load across the joints due to interlocking of the aggregate cannot be relied upon."]

Book Reviews.

"Model Analysis of Structures." By T. M. Charlton. (London: E. & F. N. Spon, Ltd. 1954. Price 21s.)

ENGINEERS who are concerned with complex statically-indeterminate plane frames and who are interested in model analysis will find this book useful. Whether or not to use model analysis is rather like the choice of a method of mathematical analysis, and depends to a large extent upon the engineering background of the designer. Many designers consider that there are few plane frames for which model analysis is a necessity, or that a plane frame so complex that the usual methods of analysis are unmanageable is possibly a bad structure from an engineering point of view, having in mind thermal movements, settlement of foundations, and other phenomena equally difficult to assess quantitatively.

For three-dimensional structures model analysis would appear to have many uses, but this application is not dealt with in the book. Nevertheless the author gives a great deal of information, hitherto available only in journals, particularly regarding flexural similarity and the choice of scales for models. The application of the methods to practical problems is clearly described. There is also a description of a device for measuring the internal forces in a model and which allows for the creep of the material of which the model is made.—J. E. G.

"Schuttbeton aus verschiedenen Zuschlagstoffen." By A. Hummel and K. Wesche. **"Die Ermittlung der Kornfestigkeit von Ziegelsplitt und andern Leichtbeton-Zuschlagstoffen."** By A. Hummel. (Berlin: Wilhelm Ernst & Sohn. 1954. Price 7 D.M.)

THE first report describes tests made to ascertain the cement content necessary to produce lightweight concrete placed in situ with compressive strengths of 285 lb. and 427 lb. per square inch. The results were as follows for concretes with various aggregates (in lb. of cement per cubic yard of concrete). For a strength of 285 lb. per square inch, using cement with a compressive strength of 3200 lb. per square inch at 28 days: Gravel and broken brick, 168; broken limestone and blastfurnace slag, 135; clinker, 252. For a strength of 285 lb. per square inch, using cement with a compressive strength of 4620 lb. per square inch at 28 days: Gravel and broken brick, 135; blastfurnace slag, 101; broken brick, 168; pumice, 295; clinker, 252. For a strength of 427 lb. per square inch, using cement with a strength of 3200 lb. per square inch at 28 days: Gravel and broken brick aggregate, 211; limestone and blastfurnace slag, 168. For a strength of 427 lb. per square inch using cement with a crushing strength of 4620 lb. per square inch at 28 days: Gravel, broken limestone, and blastfurnace slag, 168; broken brick, 211; pumice, 336; clinker, 295. The size of the aggregate was between ¼ in. and ⅛ in.

The second part of the report describes compressive tests of various aggregates.

"Spanbeton" (Entwicklung, Konstruktionen, Herstellungsverfahren und Anwendungsgebiete). By Hans Möll. (Stuttgart: Berliner Union. Price 48 DM.)

THIS book (of 188 pages) in the German language is a collection of practical applications of prestressed concrete. Various methods of prestressing are reviewed dating back to Lambot's patent in France in 1855. A brief review of prestressing in most countries is given, including some outstanding examples. The author describes in detail practical problems of prestressing precast floor beams, continuous beams, frames, shell roofs, tanks, pipes, spiral staircases, highways, piles, bridges, hydraulic structures, etc. Many useful details are included and references are given to the original papers on the subject. German patents relating to prestressed concrete are listed.

"A.S.T.M. Standards on Mineral Aggregates, Concrete, and Nonbituminous Highway Materials." (Philadelphia: American Society for Testing Materials, 1952.)

IN this booklet are given 93 specifications for concrete or materials connected with concrete, and the standardised methods of testing. Many of these have been accepted by the American Standards Association. Amongst them are many dealing with methods which have not been standardised in this country including methods of measuring the air content of freshly mixed concrete, methods of comparing concretes on the basis of the bond strength, the use of the flow table, the cutting of specimens from hardened concrete, and specifications for air-entraining admixtures.

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SITUATION VACANT. Engineer, with degree or equivalent in engineering, required by the British Cast Concrete Federation to assist in development work connected with concrete vibrators, etc., at research laboratories of the Cement and Concrete Association, Wexham Springs, near Slough. Applicants should preferably be under 26 years of age, but, if older, should have had experience in allied fields. Applications, giving full details, including qualifications, age, experience, and ability, also commencing salary expected, should be addressed in writing to SECRETARY, B.C.C.F., 105, UXBRIDGE ROAD, LONDON, W.5.

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SITUATION VACANT. Reinforced concrete designer-draughtsman required by Ashmore, Benson, Pease & Co., Stockton-on-Tees. Applicants should be fully experienced in designing and detailing reinforced concrete structures, foundations and other civil work. Apply, stating age, experience, etc., quoting reference D, to STAFF PERSONNEL OFFICER.

SITUATION VACANT. Reinforced concrete detailer-draughtsman required by consulting engineer, Haslemere, Surrey. Salary £500 to £700 per annum, according to ability and experience. Office five minutes from station. Furnished flat available if required. Apply, giving full details, to Box 4061, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Consulting structural engineers require assistants with at least two years' office experience in designing and/or detailing steel and reinforced concrete frameworks and foundations following Graduateship and National Service. Good pay and prospects with much varied and interesting work. Apply ANDREWS, KENT & STONE, 60 Wardour Street, London, W.1. Gerrard 9341.

SITUATION VACANT. Concrete products. Assistant general foreman required by Stent Precast Concrete, Ltd., at their Dagenham Dock Works. Pension scheme in operation. Knowledge of prestressed concrete an advantage. Please state age, salary required, and all details, to STENT PRECAST CONCRETE, LTD., 1 Victoria Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer required in civil engineering department of The Coppee Co. (Great Britain), Ltd., for colliery structures (buildings, bunkers, tanks, etc.). Duties include quantities. Staff pension scheme in operation. Write, stating age, qualifications, experience, and salary required, etc., to THE COPPEE CO. (GREAT BRITAIN), LTD., 140 Piccadilly, London, W.1.

SITUATIONS VACANT. Reinforced concrete detailers required in civil engineering department of The Coppee Co. (Great Britain), Ltd. Knowledge of quantities favourably considered. Staff pension scheme in operation. Write, stating age, qualifications, experience, and salary required, etc., to THE COPPEE CO. (GREAT BRITAIN), LTD., 140, Piccadilly, London, W.1.

SITUATIONS VACANT. Clarke, Nicholls & Marcel, consulting engineers, require in their London office, for reinforced concrete work, designers and draughtsmen-detailers. Permanent positions. Good prospects. Apply in writing to 21 WESTBOURNE GROVE, LONDON, W.2.

SITUATIONS VACANT. Structural engineers and detail-draughtsmen required by consulting engineers for some unusually important and interesting projects. These appointments may be permanent and will be well paid with expenses and allowances, including two weeks' paid holiday this year. Reply, stating qualifications and experience in reinforced concrete and steel structures, to Box 4063, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting engineers, Westminster, require senior designer for part-time work, with considerable experience of prestressed concrete bridges. Write in confidence, stating age, qualifications and details of experience. Box 4064, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Qualified civil or structural assistant engineer required for temperate climate in British East Africa. Minimum age 28. Three year's tour under contract. Post in London offered for three to four months prior to tour. Salary commensurate with qualifications and experience. Terms of contract to be mutually agreed. Full particulars to Box 4065, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

(Continued on page lii)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page* li)

SITUATIONS VACANT. AIR MINISTRY WORKS DEPARTMENT requires in London structural engineer designer/draughtsmen experienced in reinforced concrete or structural steelwork. Applicants should have sound technical training and several years of varied experience in design and detailing of (a) reinforced concrete construction for all types of buildings, or (b) steel-framed sheds, warehouses, and kindred types of buildings. Salaries up to £780 p.a.; starting pay dependent upon age, qualifications and experience. Extra duty allowance or overtime payable. Reasonable prospects of promotion. Posts temporary and non-pensionable but have long-term possibilities. Competitions held periodically to fill established vacancies. Applications from natural-born British subjects only, quoting A/E and stating age, qualifications and previous appointments, giving dates and stating type of work done, to AIR MINISTRY, S.2(h)/NA, Cornwall House, Stamford Street, London, S.E.1.

SITUATIONS VACANT. Experienced reinforced concrete engineer-designers required. Particulars of training and experience, and salary required, to F.C. CONSTRUCTION CO., LTD., City Road, Derby.

SITUATIONS VACANT. THE BRITISH REINFORCED CONCRETE ENGINEERING CO., LTD., have vacancies for reinforced concrete designers and detailers, with experience, in their Stafford, London, Liverpool, Bristol, Newcastle-on-Tyne and Glasgow offices. Staff pension scheme and five-days' week.

PROFESSIONAL SERVICES.

PROFESSIONAL SERVICES. Highly-qualified structural engineer, experienced in reinforced concrete structures, including shell structures, offers services of staff of structural designers and draughtsmen. Personnel detached to employer's office or site if required. Apply Box 4060, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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WANTED.

WANTED. Urgently required, several copies of "Chemistry of Cement and Concrete," by Lea & Desch. Good price paid. Offers to Box 4062. CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Steel fencing stakes. Chain link, wire netting, etc. List on request. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1.

FOR HIRE.

FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, Terminal House, London, S.W.1. Telephone: Sloane 5259.

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Delivering and Storing Loose Cement.

THE vehicle for delivering loose cement seen on the left of *Fig.* 1 comprises two spherical containers with a total capacity of 7 tons mounted on a lorry. The containers are of aluminium alloy and are designed for an internal pressure of 40 lb. per square inch. Each container is filled through a manhole at the top and is discharged by compressed air through a hose connected to a valve at the bottom. A compressor is carried on the lorry, and it is stated that cement may be pumped to a height of over 40 ft. The unladen weight of the vehicle and container is 3 tons.

The silo (also seen in *Fig.* 1) for storing cement has a capacity of 10 tons. It is 16 ft. 4 in. high, has a greatest width of 6 ft. 5 in., and is constructed of resin-bonded laminated wood. At the rear of the silo and extending the full height are two longitudinal members with footholds between them to allow access to the manhole and filter at the top. The two longitudinal members form a support for the silo when it is being transported in a horizontal position, and they can also be used as skids on which the silo can be moved. The hose supplying cement is connected to a valve at the rear of the vehicle, and the cement passes through a filter at the top of the silo.

Fig. 1.—Vehicle and Container.

The silo has semi-automatic equipment (*Fig.* 2) for weighing up to 200 lb. of cement and delivering it to the mixer. As the cement hopper is pushed under the discharge gate at the bottom of the silo it automatically operates the discharge valve. The cement flows by gravity into

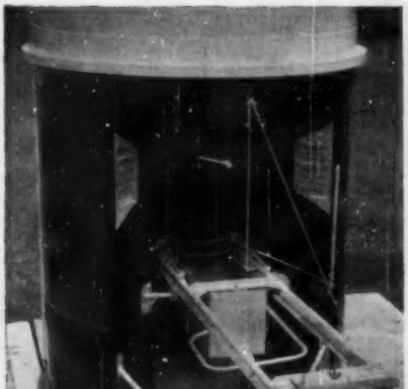

Fig. 2.—Discharge from Silo.

the hopper. The amount of cement discharged is determined by a weight sliding on an arm. The filled hopper is pulled along the track extending from the silo and is discharged, as shown in *Fig.* 1, by tilting the hopper over the skip of the mixer. The forward movement of the hopper closes the valve and stops the supply of cement from the silo. When the silo is moved the track and hopper are raised and locked to the side of the silo. The weight of the silo and weighing device is about $11\frac{1}{2}$ cwt.

The cement vehicle and silo, known as the Portasilo, are supplied by the Portasilo Co., of York.

Heating Coils in Concrete Floors.

THE hot-water heating system installed in the floors of a block of residential flats 14 stories high at Chicago, U.S.A., is described in a recent Bulletin published by the American Heating, Piping and Air-conditioning Contractors' Association. The system is designed to produce a comfortable temperature inside the building when the outdoor temperature is -10 deg. Fahr. The coils are of copper tube of $\frac{3}{8}$-in. outside diameter, and are placed $\frac{3}{4}$ in. from the bottom of reinforced concrete floor slabs 7 in. thick. Water is supplied from a tank on the roof at a temperature of 175 deg. Fahr. and returned at 145 deg. Fahr.; the average temperature in the coils is 160 deg. Fahr. About two-thirds of the heat is directed downwards through the ceiling and the remainder upwards through the floor above. It is claimed that this method is cheaper than other means of heating such buildings.

Advanced Education in Reinforced Concrete in France.

IT is reported in a recent Bulletin of the Société des Ingeniéurs Civils de France that a new college, the Institut Supérieure du Béton Armé, has been opened in Marseille. Courses of post-graduate standard will be held in the theory and practice of reinforced concrete. These courses will last one academic year and attendance will be restricted to graduates in engineering.

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